DANGEROUS STRUCTURES

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DANGEROUS STRUCTURES:

A Handbook for Practical Men.

BY

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PREFACE.

In the present volume the Author's object has been to deal with some of those awkward problems in building which demand prompt solution, to suggest ready means for getting over difficulties which frequently occur in practice, and to supply data from which efficient and at the same time economical remedies may be designed to counteract evils arising from structural defects. To this end, the results of some practical experience have been summed up and presented to the reader in a concise form, together with so much theoretical demonstration as may not be too abstruse for ready reference. Tables showing the ultimate strength of such structural accessories as lintels, beams, struts, and posts, have been supplied. These tables have been expressly calculated for the purpose, and as they have been carefully checked, it is hoped that they will prove useful and reliable as memoranda for reference by surveyors, builders, and others, and will be conducive to economy of time.

The work comprised in the following pages was first published in 1890, as a series of articles in the *Building World*, when it met with so much appreciation that it was thought advisable to embody it in its present more permanent form. All the matter in the book has been thoroughly revised for republication, and an index has been provided.

The favourable reception accorded to the Author's handbook on 'Shoring and its Application,'* encourages him to hope that the present work will commend itself to the approval of practical men.

* London: Crosby Lockwood and Son, 1887.

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DANGEROUS STRUCTURES.

CHAPTER I.

FOUNDATIONS.

The surveyor who is called in to inspect a dangerous structure, frequently find himself in a position analogous to that of a medical man in attendance upon a patient who is "in a very bad way." Great issues may depend upon the diagnosis of the professional adviser, and the remedies which he may feel called upon to prescribe. But in one sense, the experiences of the surveyor are capable of a more fundamental application than those of the physician or surgeon; for not only does it rest with him to mend or mar the ailing structure which he sees before him, but he may find in it examples of what to avoid in other fabrics which may be devised and carried out by him. Constitutional defects in buildings, unlike those of the human subject, can actually be prevented from recurring in future instances.

Undoubtedly, defects in old structures are more frequently due to faulty foundations than to any other cause. The equal distribution of weights when building upon a site has not been in the past, and is not invariably now, attended to upon scientific principles. To many persons the arrangement of foundations according to the relative compressibility of the soil is almost a new

doctrine. How frequently have we seen one portion of a building carried to a much greater height than the remainder, while yet the same spread of footings and concrete foundations is allowed to prevail throughout? The results of such experimentalising must often be an unequal subsidence in the foundations, causing dangerous dislocations in the superstructure. Time-honoured customs, thoughtlessly followed, are sometimes to blame for anomalies in construction which may be attended with disastrous results. For example, admitting what no sane man will deny, namely, that the weights of various parts of a structure should be distributed over proportionate areas of ground, why should the spread of footings be regulated by the thicknesses of walls, instead of by their height and consequent weight? In reference to footings, it may be questioned, indeed, whether brick ones are of much utility when they project more than $4\frac{1}{2}$ inches. At a projection of 9 inches, a header ceases to have any tail under the wall, and when it has a tail of anything less than half its length, it seems incredible that the presure of the footing upon the foundation can represent any significant portion of the weight above. Cases have been known in which walls have sunk, and their footings have turned up, clearly proving that the latter were unable to force their way through the soil. Had the bricks in the footings been bad, they would probably have been broken across. Failing this, what could have prevented them from turning, except the adhesive power of the mortar? Clearly, not statical conditions, which are supposed to be independent of "friction and sticktion." When a wall has sunk, and its footings have turned up, it may be asked, what is best to be done? First, then, let it be shored and needled up as necessary, and if it is to be raised and restored to its

former position, let that be done if feasible. The defective footings should be taken out, a proper excavation should be made, and a solid bed of concrete should be formed beneath the wall, so that its surface should be level with the bottom of the footings. Finally, the footings should be restored with brickwork in cement, when the concrete is set, and should be well pinned up under the wall. If the weight of the wall is considerable, it may now be asked, over what surface of concrete is it distributed? For reasons stated, it will not be safe to assume that it is evenly distributed over the whole width of the footings, if these are executed in the ordinary way; but we may assume that it is distributed over a width of $4\frac{1}{2}$ inches on each side of the wall. Ordinary concrete, made with six parts of aggregate to one of lias lime, may be safely loaded to the extent of 6 tons per superficial foot; and Portland cement concrete, made with seven parts of aggregate to one of cement, will stand from 8 to 10 tons. Of course the strength of concrete increases considerably after it has set for some months. In the case of a heavy structure, such as a factory chimney, for instance, a better distribution of the weight can be ensured by increasing the depths of the footings, laying them in double or treble courses, or by diminishing the projections of their set-offs, so that they shall spread less rapidly than in ordinary cases; but the best way to equalise the pressure upon the footings is by forming them with through slabs of Yorkshire stone. The practice of laying footings dry and afterwards grouting them with liquid cement is, no doubt, a good one. Brick footings should be all headers, and connected with hoop-iron bond, tarred and sanded.

The next point to be settled in restoring defective foundations is, how wide the concrete should spread in

order to distribute the weight of the structure over a sufficient surface of soil? This depends upon the nature of the soil. Other things being equal, it may generally be inferred that a bottom at a considerable depth is less liable to be shifty than one of the same nature near the surface. It is made compact, and retained in position by the pressure of the superincumbent earth. The lateral pressure of the surrounding soil will also frequently have considerable influence upon the foundation of a building. If there is a large building sunk to a greater depth in the vicinity, this will help to retain the earth on that side, so that subsidences will be more likely to occur in other places. A building in the south of London, close to the river side, was found to have sunk considerably on the side farthest from the river, the side towards the river having scarcely sunk at all. This was made evident by the directions of the cracks in the walls and arches of the building. The explanation was that the embankment wall of the river retained the earth on that side, and prevented the foundations from subsiding. If the foundation is of clay, much will depend upon whether the stratum runs level or not. It has been noted that the Oxford and London blue clays slip at an inclination of one in ten, one in twelve being considered safe.

With regard to the bearing power of different soils, most of the soils in and about London will safely carry $2\frac{1}{2}$ tons per superficial foot; and for ordinary houses they are rarely required to carry more. The blue till or clay of Glasgow has been stated to be as firm and solid as a rock; and yet the Townsend chimney, at Port Dundas, which is built upon this soil, without any bed of concrete, does not weigh more than $3\frac{1}{6}$ tons per superficial foot upon the foundation. The compact

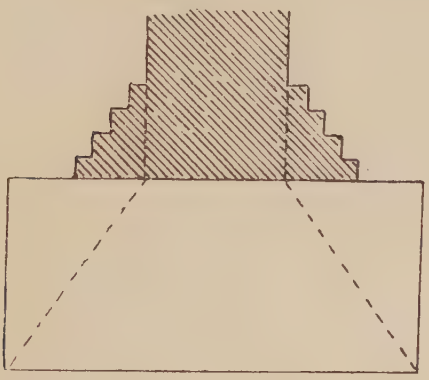
coarse gravel or ballast, which overlies the London clay for the space of about two miles on both sides of the Thames, may be loaded to the extent of 4 tons per superficial foot, or more. The chimney of the South Metropolitan Gas Company's works, weighs about .58 ton per foot upon its foundation. Tall chimneys afford a crucial test of the solidity of foundations, because the slightest inequality of subsidence would, with these structures, bring about the most disastrous results. The foundations of heavy structures should go, if possible, to greater depths than those of lighter surrounding buildings; but if a building is not very heavy, it is sometimes undesirable to carry its foundations too far down. There may be a thin stratum of clay near the surface, which is capable of bearing the required weight, and beneath this there may be a softer soil, quite unsuitable for a foundation. Once pierce the thin stratum of clay, and you may be compelled to go down another 20 feet or 30 feet in order to reach a sufficiently firm bottom. One can tell whether the stratum is thick enough to carry an ordinary building, by sounding it with an iron bar or length of piping, or a wooden pole. The soil is sounded by bumping the end of the pole or bar, held vertically upon the earth. A dull, heavy thud betokens a solid bottom, while a slightly prolonged "ploshy" sound indicates a soft soil beneath. Practice accustoms the ear to judging of these things.

If one portion of an old building is found to be sinking more than another, this may often be remedied by putting in a thick bed of concrete under the footings. The concrete, when set, will act as a species of lintel, bridging over the softer portions of the site into which the footings would otherwise sink. This should remind us that a concrete foundation is more or less subject to

a transverse strain, and it raises the question, how thick must the concrete be in proportion to its projection beyond the footings of the wall? It is obvious that if we place a very thin bed of concrete beneath a very heavy wall, making the concrete project considerably beyond the footings of the wall, the wall may sink with the concrete immediately under it, the concrete at the sides being broken off, and left behind in the downward progress of the structure. If the concrete had been thicker, it would not have broken, because the weight of the wall would have been distributed over the entire width of the concrete. The general practice is to proportion the thickness of the concrete to the height, and consequent weight of the structure. In building factory chimneys, a thickness of 1 foot 6 inches is adopted for shafts up to 40 feet high, an extra 4 inches of thickness being added for every additional 10 feet of height. But this is a mere arbitrary rule. The true question is, what is the slope or angle at which the weight can be distributed downwards? We know what it is supposed to be in the case of ordinary footings, where the projection is to the depth as three to four, which gives a slope rather steeper than 60°. I am inclined to think that the slope should be quite as steep as this; and of course the steeper it is the better. Let us now see how this would apply in practice. Let Fig. 1 be a section through the footings of a wall which is to have a concrete foundation. We will assume that we know the weight of the wall and all that it carries, and that we also know how many tons per superficial foot the soil upon which we are building will safely bear. This enables us to fix the width of our bed of concrete, which we set off centrally with the wall. The weight is supposed to be distributed first over the width of the footings, and from

thence over the width of the concrete. But it certainly is not safe to assume that the whole weight is distributed evenly over the width of the footings, and it will be better to leave the footings out of consideration; there-

Fig. 1.



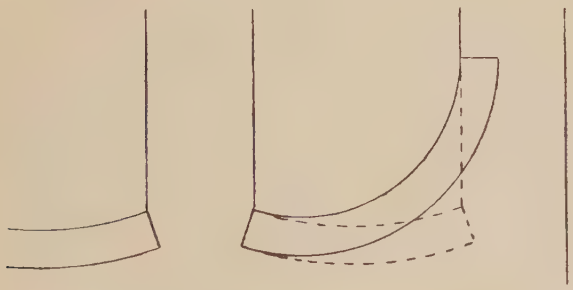
fore, draw the dotted lines from each face of the wall to the base of the footings, and from thence draw the dotted lines sloping outwards at the required angle, as shown, and these latter will determine the thickness required for the bed of concrete. If an old wall, which is beginning to sink, has concrete already under its footings, another and wider bed of concrete may be introduced underneath, either throughout the entire length of the wall, or only at places where the soil shows a tendency to subside. If new concrete is to be put in throughout the entire length of the wall, it will have to be done piecemeal, in lengths of from three to four feet at a time; and it will be best to commence operations near the middle of the wall, which is, of course, greatly supported by the adjacent parts, and proceed outwards

towards the quoins or extremities of the building, the whole of which should previously be shored up.

Everybody is familiar with the expedient of putting in inverted arches to equalise the pressure upon the foundations of a building. An ordinary application of this is in the basement storey of a high house, the piers between the windows pressing only upon a small portion of the foundation, and having to support the whole of the intermediate brickwork, except a small portion in the basement storey. The piers with their load would naturally sink, while those portions of the foundation between them, being only weighted with the brickwork around the basement windows, would scarcely sink at all, the result being the partial crushing of that brickwork, and the breaking of the continuity of the footings. To prevent this, inverted arches are introduced between the piers, which have the effect of distributing the pressure of the piers equally upon the foundation. The thrust of each arch is counteracted by the equal thrust of a similar arch opposed to it, until we come to the angle pier of the building. The pressure of the earth against this pier may suffice to counteract the thrust exerted by the arch against it, but by rights the pier itself should be sufficiently massive to overcome the thrust by its own statical resistance. Otherwise, the lower part of the pier may be pushed out of the perpendicular, and the structure may become dangerous. Let us suppose that an angle pier, under such circumstances, is being pushed out of the perpendicular, through the portion of the arch next to it having become partially flattened, so as to thrust the pier out; what, it may be asked, should be done? First of all, the necessary shoring should be done, the upper part of the angle pier being upheld by needling, so that its lower part can be rebuilt. The

pier and the defective arch can then be reconstructed; but how are we to prevent a recurrence of the disaster? There is no doubt that a sufficiently deep bed of concrete under the footings would act as a lintel, and do the work that would otherwise be done by the inverted arches. Another way would be to increase the statical resistance of the angle pier by adding to its weight, just as the mediæval builders increased the resistance of their buttresses by weighting them with pinnacles. It might be inconvenient as well as expensive to carry up an erection in the nature of a chimney stack above the

Fig. 2.



pier merely for the purpose of adding to its weight. The best thing to do would be to increase the width of the pier. This, which, probably, need only be done in the basement storey, would not only increase the weight of the pier, but would reduce the span of the arch in proportion to its rise, thereby giving the thrust more of an upward direction, as well as shifting the centre of the arch inwards. If it should be impracticable to widen the pier, Fig. 2 shows a suggestion for meeting the difficulty, the original form of the arch being indicated by dotted

lines. There is yet another expedient for preventing the spread of inverted arches, and that is by the introduction of iron tie-rods, passing right through the brick piers, and being secured to them by means of large iron washers. These rods should be inserted at the springing of the arches; and they furnish us with a ready means of checking the spread of a dangerous movement which has commenced.

Old buildings are found in some parts of London having horizontal balks of timber laid beneath the footings of their walls. These timbers, as might have been anticipated, become rotten through being saturated with wet, and allow the walls above them to subside, in many cases unequally. In such cases, when the mischief has not gone too far, and when the building is worth saving, the whole structure should be shored, and the timber cut out in lengths of two or three feet at a time, and replaced by good concrete. These lengths of concrete should be got in as soon as possible, with intervals between them, and the concrete should be allowed to set before the intermediate portions are disturbed. As for the amount of timber which can with safety be removed at a time, that will depend upon the condition of the brickwork above, and over how wide a span it is capable of sustaining itself without support. It may be useful, in similar cases, to cut out the timber in lengths of a foot or eighteen inches, at intervals of two or three feet or more, carefully to wedge up to the footings with slabs of stone where the portions have been removed, then to remove the intermediate portions of timber, and finally to fill in the excavation with concrete, the slabs of stone being allowed to remain, and becoming imbedded in the concrete. In all such operations, when a wall has partially sunk, the tendency of concrete to expand in setting

may be of service, in conjunction with shoring and other means to be noticed hereafter, to assist in restoring the wall to its proper position. A mixture of hydraulic and common lime will cause an expansion during the process of slaking, which frequently does not complete itself until after the concrete is laid. Blue lias concrete will, in hot weather, expand as much as one-thirtieth of its bulk, and in frosty weather, about one-fiftieth. Perhaps there will be no harm in protesting here against the practice of throwing in concrete from a height. The practice has been repeatedly condemned before, and yet many builders persist in following it. It ought to be evident that when ingredients of different specific gravities are mixed, the effect of gravitation upon them must be to send the heavier constituents to the bottom ; therefore, the less concrete is exposed to the effect of gravitation before it is set, the less liable is it to become unmixed by the precipitation of the heavier portions of its aggregate.

CHAPTER II.

WALLS AND PIERS.

The conditions which govern the strength respectively of walls and isolated piers are somewhat different. If the brickwork or masonry of a wall does not crack immediately under a point where great vertical pressure is applied, it is improbable that any failure will occur lower down. To explain this, let us suppose the common case of a heavily-loaded girder whose end is built into a wall, without any sufficient stone template under it to distribute the pressure over the requisite extent of brickwork. The probability is that the brickwork immediately under the end of the girder will crack, the cracks extending outwards and downwards, and gradually dying away. The bonding of the brickwork enables the pressure to distribute itself over an area of walling which becomes wider and wider as we go further down; so that if we can only provide for a sufficient distribution of pressure immediately under the girder, we may set our minds at rest regarding the safety of the brickwork down to the footings. Apart from this spreading tendency of the lines of pressure, any quantity of brickwork forming part of a continuous wall is stronger than the same quantity in an isolated pier, because in the former case it is sustained by the lateral pressure of the surrounding work. The importance of this lateral sustaining power is often very great in old walls, which may stand for indefinite periods if left alone, but which often crumble rapidly away when a

few bricks are hacked out, or even when the pointing is disturbed.

When an isolated pier has been overloaded, the case is altogether different, and, it may be said, much more dangerous than that of a wall. The pressure cannot extend itself over a wider mass of brickwork, but it is augmented, as we go lower down, by the weight of the pier itself, so that cracking or crushing is most likely to occur near the footings. Hence, the work in sight may be perfectly sound, while the part that is failing may be out of view and difficult of access. When a pier is overloaded, it will probably give warning by sinking. If the sinking takes place equally, that is, without deviation from the vertical, there is reason to hope that it may be due only to a subsidence of the foundation, but if unequally, it may be due to a partial crushing of the brickwork. In either case, it will be advisable to relieve the pier of its load. The entire load should be sustained by shoring of sufficient strength, resting upon foundations which will not be disturbed by the necessary excavations. A few light raking shores may be necessary to sustain the pier while the earth around it is being dug out to admit of an inspection of its lower part. Probably the entire pier will have to be rebuilt or replaced by an iron column. A safe limit of 8 tons per superficial foot is often quoted in engineers' tables for the load upon good stock brickwork in mortar, and 10 tons per foot is not too much for the best Staffordshire brickwork in cement. The greatest pressure upon the brickwork in the Townsend chimney at Glasgow is $8\frac{1}{3}$ tons per foot, and that upon the South Metropolitan Gas Company's chimney about $7\frac{1}{4}$ tons. A common stock brick commences cracking under a pressure of 10 tons, and as the area of a brick is little less than one-third of a foot, this does not allow much more than

30 tons per foot for the ultimate strength. Taking the safe load at one-fourth of that required to produce fracture, it amounts to from $7\frac{1}{2}$ to 8 tons per foot. Doubtless the mortar-joints form an element of weakness, but from experiments upon brick piers not exceeding twelve diameters in height, the load required to produce fracture has been proved to be 29 tons per superficial foot upon brickwork in mortar, and 30 tons upon brickwork in cement. As regards the strength of Staffordshire bricks tested singly, the common ones are about one-and-a-half times as strong, and the pressed ones about twice as strong, as ordinary stocks. But a great deal of the brickwork in and about London is far inferior to that built with good stocks and good mortar. Not only are the bricks used of a bad quality, being soft and friable, but their irregular shapes conduce to fracture by concentrating the pressure upon a few points only of their surface. Added to this, the bad mortar used with them does not fulfil its purpose in binding the work together, or in equalising the pressure upon the beds, because the greater portion of it, being improperly compounded, has not the property of setting, but becomes and remains a dry crumbling mass. Brickwork of this kind will not safely carry more than 5 tons per superficial foot, and when loaded, as it occasionally is, to the extent of 12 tons, symptoms of danger soon begin to display themselves.

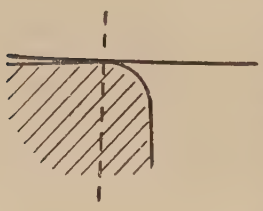
As regards the strength of good masonry, we know that the Portland stone piers in St. Paul's Cathedral carry 17 tons per superficial foot; the red sandstone pillar in Elgin Chapter House, rather more; and the coursed rubble piers in St. Peter's, Rome, about 15 tons. From experiments upon stone piers about twelve diameters high, it is found that rubble masonry, flat bedded, in lias lime and sand, begins to crack at 40 tons per foot, while

similar work with irregularly-shaped stones shows fracture at only 15 tons. The latter is about half as strong as good brickwork in mortar, and the former possesses about the same strength as the best brickwork in cement.

In repairing dangerous structures, the strength of stone templates and bedplates becomes a matter of great importance. It is of the highest importance that a stone template shall lie upon a level bed, and that that bed shall continue to be level. Suppose, for example, that the end of a girder has been built into a wall without a proper template, and that the brickwork beneath it has become cracked or partially crushed. The girder, with its load, must be supported by shoring, the defective brickwork cut out and made good with new work in cement, and a proper template pinned in under the end of the girder. Suppose, now, that the weight is allowed to come upon the template before the cement in the new work has set. The effect will be to compress and squeeze out the cement under the centre of the girder, jamming the bricks closer together under the middle of the template, so that the latter will lie in a hollow bed and be deflected like a beam. A very slight deflection of this kind will cause the stone to crack from its bed upwards. Of course the weight should be sustained by shoring until the cement is thoroughly set. Another danger to which the stone is exposed arises from the deflection of the girder, which, by concentrating the pressure upon the front edge of the stone, may cause it to split or spall at that particular place. It may be urged that, unless the girder is cambered, some deflection, however slight, is sure to take place, and, in view of this, it might be well to give the upper surface of the stone a slight inclination towards the span; but at any rate, the worst evil may be

obviated by rounding the front edge of the stone, as shown in Fig. 3, for then, even if the pressure should be concentrated in one place, yet the line of pressure, being normal to the curve of deflection, is sure to fall within the section of the stone.

Fig. 3.



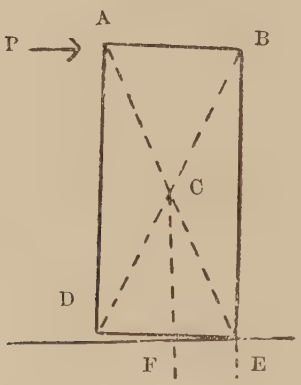
Before putting in a template, the pressure which is to come upon it should be calculated, which may be done by means of very rough measurements of the walls, floors, and roofing sustained by the girder, reckoning all walls at one cwt. per cubic foot, ordinary floors at 140 lb., and roofing at 50 lb. per square foot, the latter including all allowance for exceptional stress of weather at times. Having ascertained the maximum pressure brought to bear upon one end of the girder, the next point to consider is the superficial extent of its bearing. Some girders are very narrow in the flange in proportion to their depth and consequent load. Suppose the bearing of the girder to be 12 inches, and the width of its bottom flange 6 inches, then it will have to rest upon half a superficial foot of stone. Suppose the entire load upon the girder to be 60 tons, then 30 tons of this will be thrown upon this half superficial foot of surface. In Yorkshire stone, fracture commences under a pressure of 100 tons per foot, and it would certainly be imprudent to place 30 tons upon a surface where only 50 tons would cause fracture. One-fourth of the fracturing pressure would be a safe limit, and therefore granite, which will resist 300 tons per foot, would be amply strong for the purpose; or if Yorkshire stone were used, the girder should rest imme-

diately upon a cast-iron bed-plate, 1 foot by 1 foot 3 inches, which would distribute the pressure over $1\frac{1}{4}$ superficial foot of stone. The size of the template will, of course, depend upon the number of tons per foot the brickwork below is capable of sustaining. Upon average good brickwork, 2 feet by 3 feet will probably suffice; but of course it would be preferable for the bearing of the girder to extend over the whole width of the stone measured from the face. With regard to the depth of a stone template it could be wished, in the interest of sound building, that prevailing customs were less parsimonious. We should like to see the depth of such a stone not less than half its length. This, however, would certainly be considered extravagant, and we must usually content ourselves with a depth of one-third of the length—less than which ought not to be sanctioned. We can only hope that no stone of such dimensions will ever be subjected to a transverse strain. It would appear that a stone about 12 inches square and 4 inches thick, if supported only at its edges, would not stand much more than half a ton upon its centre. The transverse strength varies as the square of the depth, so that a 6-inch stone is more than double as strong as a 4-inch one.

With regard to the stability of walls and piers, this depends on a simple application of the theory of the bent lever. In Fig. 4, let A B D E represent a pier which is going to be overturned upon its angle E by a horizontal force P. The weight of the pier is conveniently assumed to be concentrated at its centre of gravity C, and in overturning the pier, its weight has to be lifted by leverage. The fulcrum of the lever is at E. The length of the long arm of the lever must of course be measured at right angles to the direction of the power P, so that it is equal to the vertical height of P above the fulcrum

E. The weight of the pier acting vertically, the short arm of the lever is the horizontal distance between the direction of the weight C F and the point E. The moment of stability is the weight multiplied by the distance F E, and the moment of the thrust is the power P, multiplied by its height—the forces and distances being, in both cases, expressed in the same terms. So long as the moment of the power P is less than the moment of stability, the pier will not be overturned, and

Fig. 4.



if P should cease to act, the pier would return to equilibrium, so that F E would be equal to half D E. Thus we see why the momentof stability of a wall is ordinarily stated to be equal to its weight multiplied by half its thickness, and also we see why, when P acts lower down, it has less effect upon the stability of the pier. When the diagonal A E is vertical, F E is nothing, and the pier has no stability. The question, at what inclination from the vertical does a wall or pier become unstable?

is therefore unmeaning, for it is obviously possible for a thick wall or massive pier to lean over considerably, and yet be stable, while a wall or pier of attenuated proportions may become unstable at a very slight inclination.

When a wall inclines from the vertical, the first thing to do, in judging whether it is dangerous or not, is to ascertain if all its joints are in a sound condition. If there is no danger of the brickwork or masonry sliding upon its beds, the wall may be safe. The next thing is to ascertain whether the centre of gravity of the wall's section overhangs its base. If it does not, the wall may be safe. Next, we should endeavour, approximately, to ascertain the moments of any thrusts tending to overturn the wall, and the counterpoising effect of cross or end-walls bonded into it, carefully noting, also, whether there is any tendency to draw the bond, such as may be indicated by the rupture of wall-paper or plaster. When all these circumstances have been duly considered, the surveyor should be prepared to decide whether the wall must be rebuilt, or whether it will suffice to erect buttresses against it. Setting aside the question of thrusts, let us assume the case of a 9-inch boundary wall, 12 feet high, and 20 feet long, the top of which projects 6 inches beyond its base. Its weight is just 9 tons, and, if it were vertical, its moment of stability would be $180 \times \frac{3}{8} = 80$ foot-cwts. As it is, the centre of gravity is only $1\frac{1}{2}$ inch instead of $4\frac{1}{2}$ inches within the fulcrum, the moment of stability being only $180 \times \frac{1}{8} = 22\frac{1}{2}$ foot-cwts." To restore the original stability of the wall, we must increase its moment by $57\frac{1}{2}$, or say 60 in round numbers. This could be done by means of four buttresses, spaced equidistantly, and each having a moment of 15. Suppose each buttress

to be carried to the full height of the wall, to be of triangular form in side elevation, and projecting 2 feet at the base, so that its centre of gravity would be at 1 foot 2 inches measured from its extreme projection inwards; then its weight would have to be 13 cwt., which would necessitate its being 1 foot 2 inches wide. The amount of brickwork in these buttresses would be just over one-fourth of that contained in the wall.

In calculating the stability of a wall which will enable it to resist overturn, we of course assume the existence of an open joint at or near the base. As a matter of fact, any thrust or wind-pressure which would overturn a wall would have to overcome the cohesive strength of the mortar in the lowest joint.

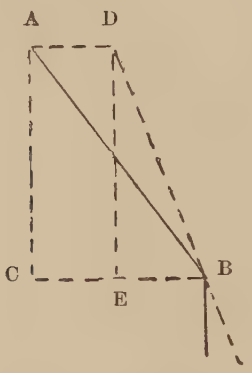
CHAPTER III.

ROOFS.

When a roof threatens to fail by thrusting out the feet of its rafters, it is convenient to calculate the amount and direction of the thrust, in order that the danger may be arrested in the most economical way. The thrust of one-half of a roof is estimated, as shown in Fig. 5, where A is the apex, and B the foot of the rafter, which rests upon a wall or pier. The latter has to sustain half the weight of the entire roof, and the stresses which come upon it. Draw A C vertically, and C B horizontally. If we assume the weight and the stresses of wind, snow, &c., to be concentrated at the apex, and to act vertically, then let A C represent one-half of the entire load on the roof, and C B will proportionately represent the horizontal thrust acting against B. This calculation will do if we have to estimate the effect of a heavy skylight or spirelet erected at the apex of the roof, but in ordinary cases the weight of the roof and the stresses upon it must be considered to be distributed over its entire surface, and for convenience in calculation, they may be assumed to be concentrated at the centre of gravity of its half-section on each side. In Fig. 5 let the point G represent the centre of gravity of this semi-section of the roof. Let the rise A C be 40 feet, the half-span C B 30 feet, and then the length of the rafter A B will be 50 feet. Assuming the roof-trusses to be 10 feet apart, then the superficial extent of roofing carried upon each half-truss

will be 500 feet. Taking the weight of the roofing, including the stress of wind and snow as 50 lb. per square foot, we have about 11 tons for the total weight to be sustained at B. This weight we assume to be concentrated at G. The pressure of A B against the opposite

Fig. 5.



rafter, where both meet at the apex, must be horizontal; therefore draw A D horizontally to represent the direction of this force. The horizontal thrust at B, in the opposite direction, must be equal to this; that is to say, either may be represented by half C B, if A C represent the weight of 11 tons. Now draw the vertical through G, from D, to cut C B in E; and D E, which is equal to A C, represents, in magnitude and position, the weight of 11 tons acting upon B, while E B, equal to A D, or to half C B, represents the horizontal thrust against B, which is just equal to $4\frac{1}{8}$ tons. The forces acting upon B are therefore two, viz., the downward force D E, and the outward E B. These two forces, compounded together, produce the diagonal thrust repre-

sented by joining D B. This line D B, produced beyond B, shows the angle at which a shore should be placed in order to resist the thrust of the roof, supposing that the static resistance of the wall were not to be taken into account.

Let us assume that the wall is of stone, 3 feet thick, and that the point B is 18 feet above the ground. We should then have 540 cubic feet of stone to each roof-truss. But the spaces occupied by windows ought to be deducted from this, besides which, if the thrusts are concentrated at the trusses, the intermediate walling can be of little service in resisting it. Probably we should not be safe in assuming that the resistance exceeds that of a pier 5 feet wide at each truss. The weight of such a pier, taken at 140 lb. per cubic foot, would be $16\frac{3}{4}$ tons, and its moment of resistance would be $16\frac{3}{4} \times 1\frac{1}{2} = 25\frac{1}{4}$ foot-tons. The moment of the thrust E B would be $4\frac{1}{8} \times 18 = 74$ foot-tons. Hence we see that, unless the roof were tied, it would overturn its abutments.

It may be asked, how such a structure could be supposed to stand at all. But it must be remembered that the allowance made for stress of weather applies only to exceptional periods, and that at ordinary times the moment of the thrust would probably not exceed one-third, or about the same as the moment of resistance. In overturning the wall it would also have to overcome the adhesive resistance of the mortar, which would probably have a moment of 12 foot-tons. Such a structure might, therefore, stand for years in a sheltered situation. But if the mortar near the bottom of the walls should have become partially loosened, and if a violent gale of wind should suddenly occur, the whole might collapse.

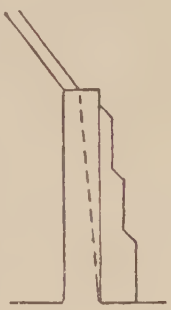
Having ascertained the magnitude and direction of the thrust D B, Fig. 5, and the weight of the pier at B, let us now find the resultant of these two forces. In Fig. 6, let the triangle D E B be redrawn precisely as in Fig. 5, so that D E shall represent the weight of 11 tons, and E B the thrust of $4\frac{1}{3}$ tons, the diagonal thrust D B being about equal to $11\frac{3}{4}$ tons. Draw the vertical F B and make it proportionately represent the weight of the pier, viz., $16\frac{3}{4}$ tons. To find the resultant of F B and D B, complete parallelogram H F D B, and join H B, which represents the required resultant. This line, H B, produced below B, shows the direction in which a raking shore should be placed, in order to assist the pier to withstand the thrust of the roof.

Fig. 6.

A raking shore or temporary buttress resists by its direction, simply conveying the thrust to the ground, its own weight being practically inappreciable. A permanent buttress of stone or brick resists by means of its weight, as explained in the previous chapter (p. 19), but it can also resist by means of its direction, if it be of sufficient projection to allow the line of diagonal thrust to fall within its outline. It would require a buttress of considerable size to neutralise the thrust of the roof in the case we have assumed. The moment of the thrust

being 74, and the moment of resistance of the pier only about 25, we should require a buttress having a moment of about 49 foot-tons, so that if its centre of gravity projected 3 feet beyond the face of the wall it would have to weigh more than 16 tons, or to contain about as much masonry as the pier it is required to support. A buttress 2 feet 6 inches wide, projecting 6 feet, and carried back by set-offs to the top of the wall, would answer the purpose and be enormously extravagant, whereas a buttress projecting 3 feet and weighing 4 tons would contain the line of thrust, while its weight would make the angle of thrust still steeper. Such a buttress is shown in Fig. 7, and as it is assumed to be 2 feet 6 inches thick, its weight will be quite 4 tons. The diagonal thrust H B (Fig. 6) having been set up in conjunction with a vertical line proportioned to this weight of 4 tons, and the parallelogram having been completed and its diagonal drawn, this latter represents the resultant, which falls at the angle indicated by the dotted line in Fig. 7, lying just within the thickness of the wall.

Fig. 7.



It will be seen that if the buttress is properly bonded to the wall, we may consider both as one mass, so that if the whole should be overturned it would turn upon the outer face or toe of the buttress. First, then, we have the weight of the wall, or rather this portion of it, increased from 16_4 tons to 20^3 tons, by means of the 4 tons added in the form of a buttress. Then we have the distance from the centre of gravity of the wall to the

fulcrum increased from $1\frac{1}{2}$ to $4\frac{1}{2}$ feet, so that the moment of resistance of the entire mass is $20\frac{3}{4} \times 4\frac{1}{2} = 93\frac{1}{4}$ foot-tons.

If the buttress were merely built against the wall with a straight joint, as is sometimes done, it would not be safe to calculate its stability as above, but the weight of the buttress would have to be sufficient to add its own separate stability to that of the wall, as explained in the previous chapter. A buttress is sometimes built against a wall without being bonded to it, because it is feared that the main structure, being much heavier than the buttress, might sink more, thus causing the bond to be broken. In a case such as we have assumed, when a roof has no ties, it is always prudent to introduce temporary ones while the structure is undergoing repair. Iron tie-rods offer a ready means of guarding against dangers arising from ill-constructed roofs. The practice of cambering tie-beams still prevails largely amongst carpenters, in spite of all that has been urged against it by Tredgold and others. When a cambered beam settles it has a tendency to thrust out the walls. If there is danger, the centre of the beam should be temporarily supported from below, while iron tie-rods may be introduced from wall to wall, or buttresses built outside.

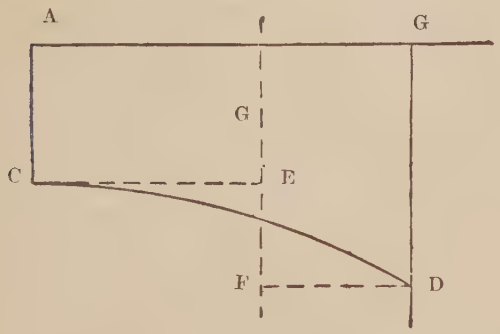
A wooden curved rib, which shows signs of splitting, may be strengthened by having a wrought iron plate bolted against its side. Cast iron has been used in such a case with success. It is more rigid than wrought iron, and quite capable of resisting any shearing stress likely to come upon it.

CHAPTER IV.

ARCHES.

The method of ascertaining the direction of the thrust of a roof, as illustrated in Fig. 5, has been applied, in Hurst's edition of Tredgold's "Carpentry," to one-half of a polygonal roof-truss, on the assumption that each half of the truss would behave as one mass, both being, as it were, hinged together at the apex. The same principle can be applied to ascertaining the thrust of an arch.

Fig. 8.



Thus, in Fig. 8, let A B C D represent one half of an arch and the mass which it supports, the point G being the centre of gravity of the mass. Draw the vertical through G, as shown, and draw the horizontal line from C, meeting this vertical in E. From D, the springing,

draw the horizontal line, meeting the same vertical in F. Now, let E F, which is equal to the rise of the arch, represent the weight of the mass A B C D; then E C will proportionately represent the horizontal thrust which each side of the arch exerts against the opposite one, and F D will represent the horizontal thrust of the arch against its abutment. In other words, let w be the weight of the mass, t the horizontal thrust against the abutment, r the rise of the arch, and d the distance of the centre of gravity of the mass from the abutment; then as

$$r : d :: w : t, \text{ and } t = \frac{w\,d}{r}$$

In Fig. 8, if we join E D, then E D will represent the amount and direction of the thrust of the arch, which of course follows the tangent of the curve.

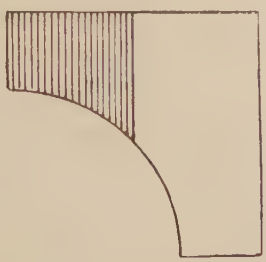
This method affords us a ready means of calculating the thrust of an arch, and of deciding whether its abutments are sufficiently massive for security. The position of the centre of gravity of the mass can be fixed quite nearly enough by the eye; indeed, if we generally assume d to be equal to one-fourth of the span, we shall not go far wrong, and shall err upon the safe side. A similar method may be pursued in ascertaining the thrust of a brick or stone vault. In the case of a semicircular vault, d would be equal to about one-seventh of the entire span. These methods are, of course, only approximate, but will be found sufficiently accurate for ordinary practice.

A semicircular vault fails usually by opening at the soffit of the crown, and at the extrados, midway between the crown and the springing. These are, therefore, the

points at which a vault may be strengthened by cramping, and the cramps should be inside at the crown, and outside at 45° above the springing. A horizontal tie-rod inserted at between four-sevenths and five-sevenths of the height from the springing to the crown may save a vault from failure. Ties lower than this may be useful, but rarely above it. An excellent way to strengthen a vaulted roof is to carry up the side walls to about 45° above the springing, and fill up the haunches with good concrete. The concrete should be filled in slowly, say to a height of about 2 feet each day, otherwise it may injure the vault by expanding.

A semicircular arch, having a solid mass above it, which finishes at a level, like that shown in Fig. 8, is subject to very simple conditions. According to Rankine, the horizontal thrust of one-half of such an arch is approximately equal to the weight supported between the

Fig. 9.



crown and that part of the soffit embraced by an angle of 45°; that is to say, the thrust is nearly equal to the weight of the portion shaded in Fig. 9. But we must not forget that in most cases an arch does not have to sustain the weight of the brickwork or masonry above it for

any considerable height, a large portion of the weight being transferred to the piers by a species of corbelling which necessarily results from the bonding of the wall. Therefore arches, like lintels, support little more than the triangular mass of brickwork which is immediately above them.

Semicircular or eliptical arches have a tendency, when failing, to sink at the crown and rise at the haunches, while pointed arches usually tend to rise at the crown and to fall inwards, or become flattened at the haunches. Hence the importance, when shoring up an arch, of supporting its crown, if it belongs to the former class, and of strutting out the haunches, if it belongs to the latter class.

CHAPTER V.

LOFTY STRUCTURES.

Structures of considerable height in proportion to the superficial extent of their foundations offer some difficult and delicate problems for solution when they show a tendency to lapse into a dangerous condition. A deviation from the vertical which, in the case of a house, may be of little consequence, may involve absolute destruction in the case of a tall factory shaft. The effect of wind becomes a highly important consideration in such cases. In factory shafts, and even some stone spires, a visible deflection takes place under strong wind, which proves that there is a certain amount of flexibility about such materials as brick and stone. It has been stated that the Townsend chimney, at Glasgow, oscillates to the extent of 3 feet, that is to say, 18 inches on each side of the vertical. The chimney is 454 feet high, and the oscillations have been observed from a distance through a telescope, having a vertical thread or wire inside it. The amount of oscillation may be measured with tolerable accuracy in such cases by counting the number of bricks, or half bricks, which mark the variation of any point from the vertical. If the oscillations take place equally on both sides of the vertical, the shaft may be considered safe, for it will maintain a vertical position in calm weather. A little oscillation of this kind is considered to be a good sign, the chimney behaving on the same principle as reeds, which bend to

the wind but do not break. If there were no oscillation at all, we should conclude that the wind-force must be expended somewhere, and that it would probably take effect at the weakest point, so that a sudden bend might occur some time, under exceptional stress of weather. If the oscillations occur in one direction only, or more in one than in the opposite direction, the chimney is certainly in a dangerous condition. If plumbed in calm weather, it will probably be found to deviate from the vertical; and this deviation may be expected to increase, unless measures are taken for remedying it. In observing the oscillation of a chimney, it is advisable to notice whether it assumes a slight curve in departing from the vertical, or whether the greater portion remains straight, the shaft bending, so to speak, at one point. The question is an important one, and it requires some careful observation to arrive at the truth. A curved deflection shows that the resistance is equally distributed, and the shaft structurally sound; whereas, bending at one point only is a sure indication of weakness at that point.

Some attention should be given to the leverage exerted by the wind upon the opposite side of a structure against which it blows. If we consider the wind-force to be directed against a single point in the height of a shaft, we can then estimate its effect at any given point lower down on the opposite side. But the wind-force is distributed over the entire portion of the chimney exposed to its action, and this is equivalent to its being concentrated at the centre of gravity of the surface acted upon. In the case of a parallel shaft, this point would be half way up the height, and in the case of a tapering shaft, this point would be a little lower. To find the height of the centre of gravity of the tapering surface, measure up two-thirds of the height of the shaft, and

divide the upper third into two parts, so that the upper part shall be to the lower as the diameter of the shaft at the base is to the diameter at the top. The point where this division is made is the centre of gravity required. Suppose a shaft to be 100 feet high, the diameter at the base being 10 feet, and that at the top 5 feet 10 inches. The diameters are as 12 to 7, and one-third of the height, or 33 feet 4 inches has to be divided into 19 parts, of which 7 must be added on to the lower 33 feet 4 inches, so that we have 12 feet 3 inches added to 33 feet 4 inches—equal to 45 feet 7 inches—for the height of the centre of gravity above the base. The leverage with which the wind force acts upon any joint in the structure is represented by the height of this centre of gravity above such joint.

The greatest wind-force acting against a vertical structure in this country is not likely to exceed 40 lb. per square foot of surface, although as much as 55 lb. has been registered on rare occasions and in exposed situations. Assuming the wind to act at 40 lb. per foot against a shaft of the above-named dimensions and square upon plan, the total wind-force expended against one face of the shaft would be a little over 10 tons. Acting upon a joint just 7 inches above the base, this force would have a moment of 450 foot-tons. Dividing this by the diameter at this point, we have 45 tons for the pressure brought to bear upon this joint by the wind, in addition to the weight already pressing upon it. At 25 feet above the base, the moment would be 200 foot-tons, and as the diameter at that point would be 7 feet 6 inches, the actual additional pressure caused by the wind would be $26\frac{2}{3}$ tons. By such calculations we are enabled to ascertain the precise extent of the danger to which a lofty shaft may be exposed

when any portion of its brickwork shows signs of being defective.

If a shaft be circular or polygonal upon plan, the centre of gravity of its exposed surface may be ascertained by the method explained, only it is usual to reckon that the wind-force against a circular shaft is equal to half that against a square one of the same diameter. In the case of an octagonal shaft it is about two-thirds. As the amount of taper assumed in the foregoing calculations agrees with the proportions sanctioned for the construction of factory shafts under the Metropolitan Regulations, it may be useful to remember that the centre of gravity of the exposed surface in a shaft of that proportion is just $\frac{9}{20}$ of the height above the base.

A shaft which inclines from the vertical may be restored to a safe position by the method adopted more than thirty years ago in the case of the Townsend chimney, which had been deflected by the wind as much as 7 feet 9 inches from the vertical, losing several feet in height, which were regained when it was straightened. The deflection commenced at about 100 feet above the base, and seems to have followed a curve. The remedy was to make a series of horizontal saw-cuts, at twelve successive heights, on the side opposite to the inclination, the shaft gradually returning to the vertical by its own weight. A shaft at Pitchcombe, Gloucestershire, being 132 feet high, was deflected 3 feet 10 inches from the vertical, about sixteen years ago. The shaft was octagonal, and the method adopted in straightening it was to cut out one course of bricks from five of the sides, and substitute a thinner course, the intervening spaces being filled with iron wedges. The work occupied about three weeks, and when all was ready, the wedges were gradually withdrawn, and the shaft returned to the upright

by its own weight. In such operations it is important to be quite sure how much to take out in order to effect the desired result; for a slight error will make a considerable difference. For instance, in the Pitchcombe shaft, it was calculated that $\frac{1}{4}$ inch taken out from the height on the side opposite to the inclination, at 40 feet above the base, would make a difference of 7 inches in the deflection at the top. The safest plan in such a case would be to measure the total height on the convex and concave sides of the deflection, respectively, when, of course, the difference between the two heights would be the amount to be taken out on the convex side. If such measurements cannot be taken, calculations are rather apt to be misleading. It might be supposed that when a shaft has bent at one point only, the inclination of the joint, at that point above the horizon, would be equal to the angle of deflection of the axis of the shaft. But this does not necessarily follow in practice. In fact, it is difficult to fix, with any accuracy, the point upon which a shaft will pivot in returning to the vertical. It is safer to rely upon experiment. The use of wedges, which can be gradually withdrawn, is decidedly to be recommended; and when the wedges have been withdrawn sufficiently far, the work between them can be pinned up to the requisite height with tiles and cement, and the wedges may be entirely withdrawn when the cement is set, after which the spaces occupied by the wedges can be filled up with bricks or tiles and cement.

One cause which conduces to the decay of all brick structures is the gradual crumbling of the mortar in the joints. This crumbling process may be postponed by pointing the work before it has been allowed to go too far, and there is no doubt that an external rendering of Portland cement is a great protection, especially to lofty structures such as factory shafts.

A shaft may incline from the vertical through a partial sinking of its foundations. When this has occurred to a sufficient extent to cause danger, the case is serious, as the shaft will probably have to be rebuilt. It should not be forgotten that the pressure of the wind against the shaft is transmitted to its foundation on the side opposite to that exposed to the wind. It may be asked, what amount of deflection is necessary to constitute danger? We should say almost any deflection which is perceptible, it being manifest that the walls of an ordinary dwelling may safely lose stability to an extent that would be highly dangerous in a lofty shaft. The batter of $2\frac{1}{2}$ inches to 10 feet of height, and the half-brick set-offs usually made inside at every 20 feet in shafts, tend to throw the centre of gravity of each wall inwards, and when there is any tendency to fall over towards one side, that tendency is assisted by the opposite wall.

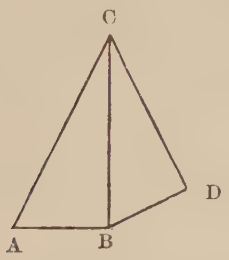
The batter of a shaft is very slight, and its effect in diminishing the pressure of the wind is not worth considering. With a spire, however, the case is different. The wind force, acting horizontally, must be resolved in a direction at right angles to the slope of the spire, and again resolved in a horizontal direction, in order to estimate its effect in tending to overturn the spire. To understand this, let a = the inclination of the slope from the vertical; P = the original force of the wind, acting horizontally; p = its resolved force, at right-angles to the slope; and n = its final horizontal effect. Then P is to p, as the secant of a is to the radius; and p is to n, as the radius to the cosine of a. Hence, we need not find p in order to find n because

$$\text{sec. } a : \text{cos. } a :: \text{P} : n, \text{ and}$$

$$n = \frac{\text{P. cos. } a}{\text{sec. } a}$$

The accompanying diagram, Fig. 10, illustrates this, the angle A C B representing the inclination of the slope from the vertical, and the triangle C D B being similar to C B A, C B being the hypothenuse of C D B, as C A

Fig. 10.



is of C B A. Then, if C A be taken to represent the original wind-force, C B is proportioned to its effect at right angles to the slope, and C D is proportioned to its final effect.

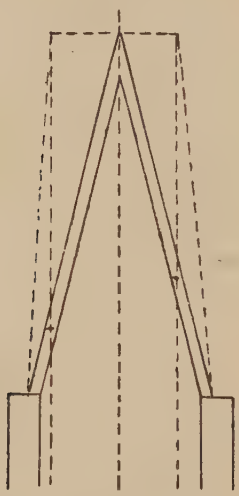
The general outline of any spire being triangular, the centre of gravity of any face of its surface is at one-third of the total height above the base. Let us assume that the wind blows with a pressure of 40 lb. per foot against one face of a square spire, 12 feet in diameter, and 30 feet high. Here the measurement on the slope is just $30\frac{1}{2}$ feet, the vertical height 30 feet, and the third proportional required, about $29\frac{1}{2}$ feet. The final effect of the wind is therefore about 38·7 lb. per square foot of surface upon a vertical triangle, whose height is 30 feet and its base 12 feet. The superficial extent is 180 square feet, and the wind-force tending to overturn the spire is $3\frac{1}{5}$ tons, which we must consider to be concentrated at the centre of gravity of the exposed surface,

namely, at 10 feet above the base. The moment of this force is, therefore, 32 foot-tons, and it is capable of overturning a wooden spire weighing $5\frac{1}{3}$ tons. If the spire were of stone, however, 6 inches thick, it could not weigh much less than 18 tons.

With a circular or octagonal spire, the same rule will apply as in the case of shafts, namely, that the wind-pressure upon a circular spire is half, and upon an octagonal spire, two-thirds, of what it would be upon a square one of the same diameter.

The thrust of a spire against the walls of the tower supporting it may be calculated on the same principle

Fig. 11.



as the thrust of an ordinary roof. Whatever the form of the spire, provided its sides be straight, the thrust increases at a regular rate towards the base, so that if it

be sufficiently tied, or otherwise held together there, there need be little fear of its giving way higher up. The thrust at the base of a spire is considerably reduced by tying it at some point above. This will be understood from Fig. 11, which shows the half-sections of two spires. The right-hand half-section shows one side of a spire which has not been tied. The weight of any triangular strip of spire is, therefore, assumed to be concentrated at its centre of gravity, namely, at one-third of the height, and the moment of the thrust is found by multiplying this weight by the horizontal distance of the centre of gravity of the section from the abutment. The direction of the thrust is also shown by a dotted slanting line. The section on the left side shows part of a spire which has been tied at a few feet above the abutment. This has the effect, as regards thrust, of concentrating the weight at the point where the tie occurs. Consequently the moment of the thrust is reduced in proportion to the proximity of this point to the abutment, while the direction of the thrust is made very much steeper, as is shown by the slanting dotted line.

CHAPTER VI.

STONE LINTELS.

Stone lintels which have become cracked so far as to be dangerous can generally be safely and expeditiously restored. In such cases, the first question that suggests itself to us is, whether there is a relieving arch over the lintel, and if so, whether the arch derives its support independently of the lintel. If the span of the arch extends as far as the extremities of the lintel, the latter can be removed without in any way disturbing the arch. In such a case, supposing the arch and the work above it to be in a sound condition, we may first strut up the opening, then remove the brick or stone spandril filling from under the arch, and then remove the lintel. After that, we can put in a new lintel, make good the jambs upon which it rests, with brick or stone in cement, and lastly, restore the spandril filling. Of course, if the arch and work above it are in a dangerous state, the whole must be pulled down from above and restored from below. If there is no relieving arch, one may be inserted before the lintel is removed. To do this, let the outline of the arch first be marked on with chalk. The work is then cut off, and the arch built in, bit by bit, after which the lintel is restored.

Arches of considerable span may thus be put in without the aid of timber centering, the existing work below being afterwards removed, according to the exigencies of the case. Sometimes a relieving arch

cannot be put in, because the abutments are inadequate to resist its thrust. In such a case, it may be necessary to substitute an iron beam for a stone lintel. Sometimes the rather extravagant device is adopted of employing a stone lintel sufficiently deep to allow of a portion being cut out at the top, so as to form a skewback at each end to receive the arch. The lintel is thus made to serve as a tie to its own relieving arch. A stone lintel is ordinarily capable of withstanding all the tensile strain which it is supposed to receive under the circumstances, and which it would receive if the whole depth of the stone were subjected to tension at once. But the strain of the arch is thrown principally upon the upper part of the stone, which may have the effect of causing it to crack at the springings of the arch. We cannot, therefore, regard this method of construction as a safe one to adopt. Sometimes it is convenient to put in a pair of stone springers, and to connect these by means of an iron tie-rod, strong enough to resist the thrust of the arch. The tie should be galvanised to prevent rust.

Before restoring a stone lintel we should, of course, feel certain that the new one is not likely to become cracked in the same manner as the original one. Such considerations may induce us to put in a stone of better quality or greater depth, or to substitute a beam of iron or of cement concrete containing a core of rolled ⊥ or I iron imbedded in it.

In calculating the strength of stone or other lintels, it is convenient to use the well-known formula applicable to beams :—

$$\frac{c\,b\,d^2}{s} = \mathrm{W},$$

where b = the breadth and d the depth, in inches ; s =

the span in feet ; c = a coefficient of strength, dependent upon the nature of the material ; and W = the breaking weight. From the comparatively small transverse strength of stone, it is convenient to express W in cwts. From the results of experiments, quoted in Gwilt's "Encyclopædia of Architecture," we obtain the following values of c, the load being assumed to be evenly distributed over the span, as it usually is in practice : Brown Portland stone, ·535 ; white Portland stone, ·4 ; burnt clay ballast concrete, ·4 ; coke breeze concrete, ·312 ; pit-gravel ballast concrete, ·285 ; Box Ground Bath stone, ·129 ; and Corsham Down Bath stone, ·096. In the concrete lintels, it is assumed that the ingredients are in the proportion of one part of cement to four of aggregate, the sectional area of the iron core being about one-eleventh of that of the lintel.

In arriving at the foregoing coefficients of strength, a considerable margin of safety has been allowed. According to Gauthey, a French authority, the value of c would be ·697 for hard limestones, and ·616 for soft limestones ; but the coefficients previously given are to be preferred, because experiments are usually made with selected materials, whereas materials ordinarily in use differ in quality. The following table shows the breaking weights of Portland stone lintels, the load being in each case distributed. The breath of the lintel is assumed to be $4\frac{1}{2}$ inches, which is the most ordinary dimension. Since the strength varies directly as the breadth, it will be easy to calculate the strength of similar lintels of greater or less breadth.

It will be observed that the strength of the white Portland lintels agrees with that of the burnt-clay ballast concrete ones with an iron core of one-eleventh of the sectional area. The core may be enlarged for additional strength.

STRENGTH OF PORTLAND STONE LINTELS.

Span in feet.	Depth in inches.	Breaking Weight in cwts.	
		Brown Portland.	White Portland.
1	4	38	28.8
1½	6	57.7	43.2
2	6	43.3	32.4
2½	8	61.6	46
3	9	65	48.6
3½	12	99	74
4	12	86.6	64.8
4½	15	120.3	90
5	15	108.3	81

The following table shows the strength of Bath stone lintels, the conditions being similar to the preceding :—

STRENGTH OF BATH STONE LINTELS.

Span in Feet.	Depth in Inches.	Breaking Weight in cwts.	
		Box Ground.	Corsham Down.
1	5	14.5	10.8
1½	8	24.7	18.4
2	10	29	21.6
2½	12	33.4	24.8
3	14	37.9	28.2
3½	16	42.4	31.5
4	18	47	34.9

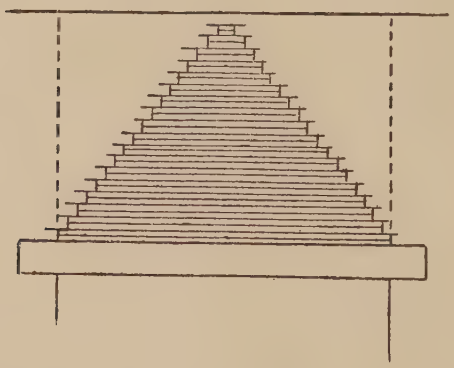
The safe load upon stone lintels may be generally taken at one-fifth of the breaking weight.

CHAPTER VII.

TIMBER BEAMS.

PRACTICAL experience is, on the whole, the safest guide in matters of construction ; but it should be supplemented by theoretical calculations, which enable us to guard against possible contingencies. For example, beams and lintels have often been put in, which, although found to be strong enough for their immediate purpose, do not

Fig. 12.



possess a sufficient margin of strength to resist the strains that may in future come upon them. Fig. 12 shows a beam which supports a mass of brickwork over an opening. As long as the brickwork remains in a sound con-

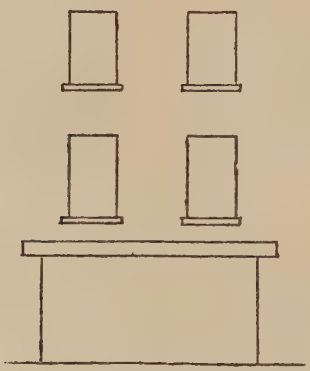
dition, the load sustained by the beam will not exceed the portion shaded in the diagram, the remaining portions of the brickwork immediately above being thrown upon the piers by a species of corbelling which necessarily results from the bonding of the work. But if the mortar should become loose in the joints, from some settlement or other unforeseen cause, the beam may be called upon to support the whole portion enclosed within the vertical dotted lines; that is to say, its load will be doubled. This is a circumstance which is liable to be overlooked, especially by speculators who build houses for the express purpose of selling them.

In calculating the strength of shop bressummers, care should be taken to allow for the manner in which the load is distributed. When the load is concentrated upon the middle of the bressummer the latter will only carry half as much as it will when its load is uniformly distributed over the whole span. The load upon a shop bressummer is often assumed to be evenly distributed, whereas the greater portion of the brickwork may be concentrated at or near the middle, as is shown in Fig. 13.

In such cases, a safe and expeditious plan, avoiding elaborate calculations, is to take the weight of the brickwork as an evenly distributed load, without deducting the openings. When closer accuracy is required, it will be convenient to remember the following particulars of stress. Let s = the stress caused by a certain load when concentrated at the middle of the span of a beam; then, if the same load be evenly distributed over the span, this will give a stress equivalent to $\frac{s}{2}$ on the middle. If the load be divided into two equal parts, and each part be concentrated at one-third of the span, the stress at each

of those points will be $\frac{4\,s}{9}$ and the total stress upon the beam will be equivalent to $\frac{64\,s}{81}$ on the middle. If the load be divided into three equal parts, one part being placed on the middle and one at each side, halfway be-

Fig. 13.



tween the middle and the supports, the stress on the middle will be $\frac{s}{3}$, the stress at each of the other points will be $\frac{s}{4}$, and the total stress upon the beam will be equivalent to $\frac{17\,s}{24}$.

To calculate the strength of timber beams by the usual formula, it is not always safe to rely upon the coefficients of strength published in books. According to Tredgold, the values of c, for beams supported at the ends and loaded on the middle are, for Riga fir,

4·73 ; for young English oak, 8·6 ; and for old English oak, 3.9. These coefficients will give the breaking weight in cwts. In Hurst's Handbook, we find a coefficient of 3.6 for white spruce fir, and 5.0 for English oak ; and in a footnote we find the following important observation : "For wood, as the experiments were made on small and selected specimens, a reduction of $\frac{1}{4}$ to $\frac{1}{2}$ should be made from the above values of c in large and unselected beams"! This would give coefficients varying between 1·8 and 2·7 for fir, and between 2·5 and 3·75 for oak. An experienced London surveyor is in the habit of taking 2.5 for fir generally ; and this is probably safe. For oak, we shall probably be safe in taking 3.25. When the load is distributed, the coefficients are 5.0 for fir and 6.5 for oak.

The following tables show the calculated breaking weights upon fir beams, the load being uniformly distributed. In the scantlings given, the depth is always stated first.

STRENGTH OF FIR BEAMS.

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 5 feet.					
3 by 3	27	$4\frac{1}{2}$ by $4\frac{1}{2}$	$91\frac{1}{8}$	$5\frac{1}{2}$ by 4	121
4 ,, 3	48	5 ,, 3	75	$5\frac{1}{2}$,, $5\frac{1}{2}$	$166\frac{2}{3}$
4 ,, 4	64	5 ,, 4	100	6 ,, 3	108
$4\frac{1}{2}$,, 3	$60\frac{3}{4}$	5 ,, 5	125	6 ,, 4	144
$4\frac{1}{2}$,, 4	81	$5\frac{1}{2}$,, 3	$90\frac{3}{4}$	6 ,, 6	216

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
		Span 6 feet.			
4 by 4	53¼	5 by 5	104½	6 by 6	180
4½ ,, 3	50⅝	5½ ,, 3	75⅝	7 ,, 3	122½
4½ ,, 4	67½	5½ ,, 4	100¾	7 ,, 4	163¼
4½ ,, 4½	76	5½ ,, 5½	138⅝	8 ,, 4	213¼
5 ,, 3	63	6 ,, 3	90	8 ,, 6	319⅜
5 ,, 4	83¼	6 ,, 4	120	8 ,, 8	426½
		Span 7 feet.			
5 by 3	54	6 by 3	77⅛	8 by 6	274¼
5 ,, 4	71⅜	6 ,, 4	102¾	8 ,, 8	365¾
5 ,, 5	89¼	6 ,, 6	154¼	9 ,, 3	173½
5½ ,, 3	64¾	7 ,, 3	105	9 ,, 4½	260⅜
5½ ,, 4	86¼	7 ,, 4	140	9 ,, 6	347
5½ ,, 5½	97⅛	8 ,, 4	182⅞	9 ,, 9	520¾
		Span 8 feet.			
5 by 4	62½	8 by 6	240	10 by 6	375
5 ,, 5	78⅛	8 ,, 8	320	10 ,, 10	625
5½ ,, 4	75½	9 ,, 4½	227¾	11 ,, 4	302½
5½ ,, 5½	96½	9 ,, 6	303¾	11 ,, 5½	416
6 ,, 4	90	9 ,, 9	455⅝	11 ,, 7	529⅜
6 ,, 6	135	10 ,, 4	250	11 ,, 8	605
8 ,, 4	160	10 ,, 5	312½	11 ,, 11	831⅞

STRENGTH OF FIR BEAMS.—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 9 feet.					
5½ by 4	67	9 by 6	270	11 by 7	470⅓
5½ ,, 5½	85¾	9 ,, 9	405	11 ,, 8	538
6 ,, 4	80	10 ,, 4	222¼	11 ,, 11	739½
6 ,, 6	120	10 ,, 5	277¾	12 ,, 4	320
8 ,, 4	142¼	10 ,, 6	333¼	12 ,, 6	480
8 ,, 6	213¼	10 ,, 10	555½	12 ,, 8	640
8 ,, 8	284½	11 ,, 4	269	12 ,, 9	720
9 ,, 4½	202½	11 ,, 5½	369¾	12 ,, 12	960
Span 10 feet.					
5½ by 4	60¼	9 by 6	243	11 by 7	403
5½ ,, 5½	77¼	9 ,, 9	364½	11 ,, 8	484
6 ,, 4	72	10 ,, 4	200	11 ,, 11	645
6 ,, 6	108	10 ,, 5	250	12 ,, 4	288
8 ,, 4	128	10 ,, 6	300	12 ,, 6	432
8 ,, 6	192	10 ,, 10	500	12 ,, 8	576
8 ,, 8	256	11 ,, 4	242	12 ,, 9	648
9 ,, 4½	182¼	11 ,, 5½	322½	12 ,, 12	864

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 11 feet.					
6 by 6	98¼	10 by 10	454¾	12 by 9	588½
8 ,, 6	174½	11 ,, 5½	293	12 ,, 12	785½
8 ,, 8	232¾	11 ,, 7	366⅜	13 ,, 5	384⅛
9 ,, 6	221	11 ,, 8	440	13 ,, 6	461
9 ,, 9	331½	11 ,, 11	586	13 ,, 7	537¾
10 ,, 5	227⅜	12 ,, 6	392¾	13 ,, 8	614⅝
10 ,, 6	272¾	12 ,, 8	523¾	13 ,, 13	998¾
Span 12 feet.					
6 by 6	90	11 by 7	335¾	13 by 6	422½
8 ,, 6	160	11 ,, 8	403¼	13 ,, 7	492¾
8 ,, 8	213¼	11 ,, 11	537½	13 ,, 8	563¼
9 ,, 6	202½	12 ,, 6	360	13 ,, 13	915½
9 ,, 9	303¾	12 ,, 8	480	14 ,, 6	490
10 ,, 6	250	12 ,, 9	540	14 ,, 7	571⅓
10 ,, 10	416½	12 ,, 12	720	14 ,, 9	735
11 ,, 5½	268¾	13 ,, 5	352	14 ,, 14	1143¼

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 13 feet.					
6 by 6	83	11 by 7	310	13 by 7	455
8 ,, 6	147¾	11 ,, 8	354½	13 ,, 8	520
8 ,, 8	197	11 ,, 11	496	13 ,, 13	845
9 ,, 6	186½	12 ,, 6	332¼	14 ,, 6	452¼
9 ,, 9	280¼	12 ,, 8	443	14 ,, 7	527⅝
10 ,, 6	230¾	12 ,, 9	498¼	14 ,, 9	678⅜
10 ,, 10	384⅓	12 ,, 12	664½	14 ,, 14	1055¼
11 ,, 5½	248	13 ,, 6	390		
Span 14 feet.					
8 by 6	137	11 by 8	328⅞	13 by 7	422¼
8 ,, 8	182¾	11 ,, 11	452	13 ,, 8	482½
9 ,, 6	173½	12 ,, 6	308½	13 ,, 13	784
9 ,, 9	260¼	12 ,, 8	411¼	14 ,, 6	420
10 ,, 6	213½	12 ,, 9	462¾	14 ,, 7	490
10 ,, 10	356	12 ,, 12	617	14 ,, 9	630
11 ,, 7	287¾	13 ,, 6	362	14 ,, 14	980

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 15 feet.					
8 by 6	128	11 by 8	322½	13 by 7	394¼
8 ,, 8	170½	11 ,, 11	430	13 ,, 8	450½
9 ,, 6	162	12 ,, 6	288	13 ,, 13	732¼
9 ,, 9	243	12 ,, 8	384	14 ,, 6	392
10 ,, 6	200	12 ,, 9	432	14 ,, 7	457¼
10 ,, 10	333¼	12 ,, 12	576	14 ,, 9	588
11 ,, 7	268½	13 ,, 6	338	14 ,, 14	914½
Span 16 feet.					
8 by 8	160	12 by 9	405	14 by 14	857½
9 ,, 9	227¾	12 ,, 12	540	15 ,, 7½	527¼
10 ,, 10	312½	13 ,, 8	422¾	15 ,, 9	632¾
11 ,, 8	302½	13 ,, 13	686½	15 ,, 10	703⅛
11 ,, 11	415¾	14 ,, 9	551¼	15 ,, 15	1054½
Span 17 feet.					
8 by 8	150⅝	12 by 9	381⅛	14 by 14	807
9 ,, 9	214¼	12 ,, 12	508¼	15 ,, 9	596
10 ,, 10	294⅛	13 ,, 8	396	15 ,, 15	993¼
11 ,, 8	284¾	13 ,, 13	646	16 ,, 12	903½
11 ,, 11	391¼	14 ,, 9	517	16 ,, 16	1204½

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 18 feet.					
8 by 8	142¼	12 by 9	360	14 by 14	762¼
9 ,, 9	202½	12 ,, 12	480	15 ,, 9	562½
10 ,, 10	277¾	13 ,, 8	375½	15 ,, 15	937½
11 ,, 8	269	13 ,, 13	610¼	16 ,, 12	853¼
11 ,, 11	369¾	14 ,, 9	490	16 ,, 16	1137¾
Span 19 feet.					
9 by 9	191⅞	12 by 12	454¾	15 by 9	532¾
10 ,, 10	263⅛	13 ,, 8	356	15 ,, 15	888
11 ,, 8	254¾	13 ,, 13	578	16 ,, 12	808¼
11 ,, 11	339½	14 ,, 9	464	16 ,, 16	1078
12 ,, 9	341	14 ,, 14	722	18 ,, 18	1534¾
Span 20 feet.					
10 by 10	250	13 by 8	338	15 by 15	843¾
11 ,, 8	242	13 ,, 13	549¼	16 ,, 12	768
11 ,, 11	322½	14 ,, 9	441	16 ,, 16	1024
12 ,, 9	324	14 ,, 14	686	18 ,, 12	972
12 ,, 12	432	15 ,, 9	506½	18 ,, 18	1458

STRENGTH OF FIR BEAMS—*continued.*

Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.	Scantling in inches.	Breaking Weight in cwt.
Span 25 feet					
13 by 13	439¼	15 by 9	405	16 by 16	819¼
14 ,, 9	352¾	15 ,, 15	675	18 ,, 12	777½
14 ,, 14	548¾	16 ,, 12	614¼	18 ,, 18	1166¼

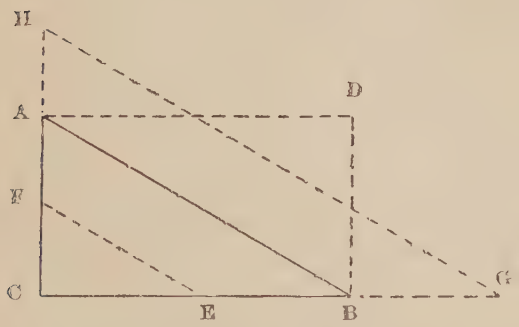
According to most authorities, the safe load upon timber beams should not exceed $\frac{1}{5}$ of the breaking weight for stationary loads, or $\frac{1}{7}$ for moving loads. It should be borne in mind that in every inhabited building there is a certain amount of vibration caused by frequent movements, and that most loads are consequently, to some extent, moving ones. By adopting $\frac{1}{5}$ of the breaking-weight as a constant coefficient of safety, and estimating the load proportionately in excess when there is likely to be much movement, we can arrive at the same result as we should by varying the coefficient of safety. Thus, for example, it is customary to calculate the load upon the floor of a public room at $1\frac{1}{2}$ cwt. per superficial foot, including the weight of the floor itself. This is a greater load than would ever be put upon the floor of a room used for purposes of entertainment; but it is by no means an excessive allowance to make for any room where dancing takes place.

CHAPTER VIII.

TIES.

Although it is true that a tie can only be strained in the direction of its length, yet the force producing that strain may act in another direction, and may be greater or less than the strain upon the tie. In the diagram (Fig. 14) let A C represent a vertical wall-face, C B a hinged flap such as is used outside a warehouse loop,

Fig. 14.



and A B a chain supporting the flap, which is hinged at C; and let the dotted triangle A D B be similar and equal to A C B. Suppose a load to be placed at B, and let D B represent the amount of the load. Then D B or A C represents the stress which would be produced upon a chain D B, supposing that the load were suspended

from D, instead of resting upon the platform or flap, and A B represents the stress upon the chain A B. If the load were placed at C we should assume it to be supported by the wall, and there would be, theoretically, no stress upon the chain. If the load were placed somewhere between C and B, the stress upon the chain would be something less than A B. Let the load be shifted to E. Draw E F parallel to A B, and cutting A C in F. Then F E represents the stress upon the chain.

We will now suppose the flap to be extended to G, beyond the point where the chain is fixed. If the load be placed at G the stress upon the chain will be greater than A B. From G draw G H parallel to A B, and cutting C A, produced, in H. Then H G represents the stress upon the chain when the load, represented by D B, is placed at G.

By this means the stresses upon diagonal ties in any direction may be ascertained, when we know the magnitudes and directions of the forces acting upon them. The method is applicable to iron ties by which chimney-stacks are secured to lower structures, to prevent them from being overturned by the wind, or to ties which are similarly employed for securing advertisement boards, or those metal frames in which the names of business firms formed with huge letters are suspended against the sky. Structures of the latter description offer little resistance to the wind.

The length of a tie does not affect its resistance, except in so far as the additional weight of metal adds to the strain. Good wrought-iron ties ought to have an ultimate tensile strength of twenty tons per inch of sectional area, but in many cases we cannot safely reckon upon more than sixteen tons. In old ties the

sectional area available for strength is often virtually reduced by corrosion, especially at the fixings, where the portions rusted may be out of sight. Thus, a $\frac{7}{8}$ inch tie may be reduced to $\frac{3}{4}$ inch in some places. In view of the tendency to lose strength by corrosion, the stress upon external ties ought not to exceed one-tenth of the ultimate strength, while it may be one-eighth or one-seventh in protected situations.

The following are the sectional areas, in inches and decimals, of circular ties of the usual dimensions : $\frac{1}{4}$ inch, ·049 ; $\frac{3}{8}$ inch, ·110 ; $\frac{1}{2}$ inch, ·196 ; $\frac{5}{8}$ inch, ·306 ; $\frac{3}{4}$ inch, ·441 ; $\frac{7}{8}$ inch, ·600 ; 1 inch, ·785 ; $1\frac{1}{8}$ inch, ·993 ; $1\frac{1}{4}$ inch, 1·224 ; and $1\frac{1}{2}$ inch, 1·764. These figures may be multiplied by the number of tons per inch which it is thought safe to allow in any special case.

CHAPTER IX.

STRUTS.

The conditions affecting diagonal struts are similar to those affecting ties. If a strut be used to resist a force acting in any given direction, the stress upon the strut will be proportioned to its length. Suppose it be required to strut up the ceiling of a room which is 10 feet high in the clear, the surplus pressure to be sustained by the strut being 10 cwt., a strut 10 feet long, placed vertically between the floor and ceiling, will be subjected to a stress of 10 cwt.; but if the strut be 11 feet long, and placed obliquely, as it must of necessity be, it will be subjected to a stress of 11 cwt.

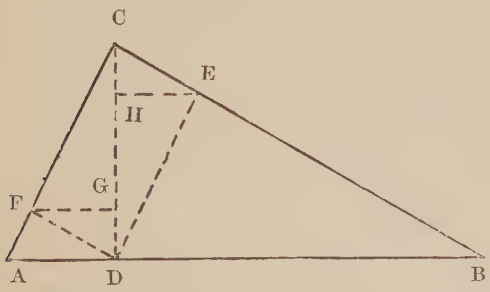
If two raking struts, at different slopes, are applied to resist a pressure at one point, the strut which is more nearly vertical will be subjected to the greater stress, and may, in certain circumstances, be called upon to sustain the whole pressure.

In Fig. 15 let A C and B C represent two raking struts, which sustain a vertical pressure at C. The vertical line C D represents this pressure, and the parallelogram of forces being completed in F and E, the distances C E and C F respectively represent the stresses upon the struts. Draw H E and F G parallel to A B, and then C H represents the proportion of the load sustained by the strut C B, and C G the proportion by the strut C A. By such means the stresses upon any number of struts may be determined, and, when these are known, we have only

to take care that the struts themselves are of sufficient strength for the purposes to which they are applied.

There are two modes of fracture to which a post or strut is liable. It may be crushed, or it may be bent until it breaks across the middle. In the majority of cases, however, a post is partly crushed and partly broken across. It is probable that no material is absolutely homogeneous, and that however carefully the pressure may be applied, there is always a tendency for the strut to bend in one direction more than another.

Fig. 15.



When a strut is of such squat proportions that it can be crushed without appreciable bending, we may consider that its strength is proportioned to its sectional area, which, in a square timber, is equivalent to the square of its diameter. When a strut bends beneath a load and afterwards breaks, it is broken by leverage. The leverage is proportioned to the deflection, which varies directly as the length and as the elasticity, and inversely as the diameter of the strut. The elasticity being proportioned to the length, it follows that the leverage of fracture

varies directly as the square of the length and inversely as the diameter; or, if l = the length and d the diameter the leverage varies as $\frac{l}{d}$. The resistance, like that of a square beam, varies as d^3, so that the leverage is to the resistance as $\frac{l^2}{d}$ to d^3, or as l^2 to d^4. Hence the strength of a strut which is long enough to be broken by bending varies as $\frac{d^4}{l^2}$.

Up to about ten diameters high, it is generally assumed that struts yield by crushing alone; and if more than thirty diameters high, it is considered that they yield by bending. Between these two proportions, it is necesary to employ calculations which take account both of crushing and bending. Wiesbach's formula, quoted by Rankine and Hurst, fulfils the required conditions extremely well. According to this formula, the strength of a strut varies as

$$\frac{d^2}{\frac{l^2}{d^2} + 1}.$$

Let us suppose the length to be extremely small in proportion to the diameter, so that the strut would yield wholly by crushing. Then the fraction $\frac{l^2}{d^2}$ would become extremely small, so that $\frac{l^2}{d^2} + 1$ would be nearly equal to 1, and the formula would be nearly $\frac{d^2}{1} = d^2$. Again, let us suppose the diameter to become extremely small in proportion to the length, so that the strut would yield wholly by bending. Then $\frac{l^2}{d^2}$ would be a large

quantity, and would be nearly equal to $\frac{l^2}{d^2} + 1$, so that the formula would be nearly $\frac{d^2}{\frac{l^2}{d^2}} = \frac{d}{l^2}$.

It will be observed that when $l = d$, that is, in the case of a cube, the formula reduces the original strength by one-half. This leaves a considerable margin of safety for short struts. When l is a large quantity, the results are within the mark according to Rondelet's experiments.

To obtain practical results from Wiesbach's formula, the following form of it is adopted :—

$$\frac{\frac{d^2 \times c}{l^2}}{\frac{d^2 \times a}{} + 1} = W.$$

Here W represents the breaking-weight, and c the absolute crushing strength per square inch of sectional area, both in tons ; l is the length, and d the diameter, both in inches ; and a is a number proportioned to the elasticity of the material. The values of c adopted are for oak 3·2 and for fir 2·5. The values of a are 250 for square or rectangular sections, and 187 for circular ones. When the post is not square the sectional area is substituted for d^2 in the upper line of the formula, and d in the lower line stands for the least diameter.

The following tables show the breaking weights for square struts of oak and fir, calculated according to the method explained :—

STRENGTH OF STRUTS.

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
	Height 8 feet.		
5	32¼	4	12
6	56¾	4½	18
7	89½	5	25¼
8	129¾	6	44½
9	178	7	69¾
10	233¾	8	101½
11	296¾	9	132¼
12	366¾	10	182½
	Height 9 feet.		
5	27¾	4	10
6	50	4½	15¼
7	80½	5	21¾
8	118½	6	39
9	164½	7	62¾
10	218	8	92½
11	279	9	128½
12	348	10	170¾
13	423¾	11	218¼

STRENGTH OF STRUTS—*continued.*

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
	Height 10 feet.		
6	44¼	4	8⅛
7	72¼	4½	12½
8	107⅘	5	19
9	151½	6	34½
10	203	7	56¼
11	262¼	8	84
12	329	9	118¼
13	403¼	10	158½
14	484¾	11	205
15	571¾	12	257
	Height 11 feet.		
7	64¾	5	16½
8	98	6	30½
9	139¼	7	50½
10	188½	8	76½
11	245½	9	108¾
12	310½	10	147¼
13	382¾	11	192
14	462½	12	223½
15	549¾	13	297½
16	641½	14	361½
17	745	15	429½

STRENGTH OF STRUTS—*continued.*

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
	Height 12 feet.		
7	58½	5	14½
8	86½	6	27¼
9	128	7	45½
10	174¾	8	69⅓
11	229¾	9	100
12	292¼	10	136½
13	362¾	11	179½
14	440¾	12	228½
15	526	13	283½
16	618¾	14	345½
17	718½	15	411
18	825½	16	483¼
	Height 13 feet.		
8	81¼	6	24¼
9	117½	7	41
10	162¾	8	63½
11	214⅓	9	92
12	274¾	10	126½
13	343	11	167½
14	419	12	214¾
15	502½	13	268
16	593½	14	327½
17	691¾	15	392½
18	797¼	16	463¾
19	909¾	17	540½

STRENGTH OF STRUTS—*continued.*

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
	Height 14 feet.		
9	108¼	6	21¾
10	150¼	7	37
11	200¼	8	57¾
12	258¼	9	84⅓
13	324	10	117½
14	398	11	156½
15	497½	12	201¾
16	568⅓	13	253¼
17	665	14	310¾
18	768¾	15	374½
19	880	16	444
20	998¼	17	519½
	Height 15 feet.		
9	99¾	6	19¾
10	139½	7	33½
11	187	8	52¾
12	242¾	9	77¼
13	306	10	108¾
14	377½	11	146½
15	456¾	12	181
16	543¾	13	239
17	638½	14	294¾
18	740½	15	356¾
19	850	16	424¾
20	966½	17	498¾

STRENGTH OF STRUTS—*continued.*

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
Height 16 feet.			
10	129¾	7	30½
11	174½	8	48½
12	227½	9	74¼
13	288¾	10	101
14	357¾	11	136¾
15	435	12	177¾
16	519¾	13	225½
17	612½	14	279½
18	712½	15	339¾
19	820	16	406¾
20	935¼	17	478½
Height 17 feet.			
10	120	7	27¾
11	163	8	44½
12	213½	9	66¼
13	272½	10	93¾
14	339	11	127¼
15	413¾	12	167
16	496½	13	212¾
17	586¾	14	264¾
18	684¾	15	322¼
19	790½	16	387¾
20	903¾	17	458½
		18	535¾

STRENGTH OF STRUTS—*continued.*

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
	Height 18 feet.		
11	152¼	8	40¾
12	200½	9	61
13	257	10	87¼
14	321¼	11	119
15	393¾	12	156¾
16	473¾	13	200¾
17	561¾	14	251
18	658¼	15	307½
19	761½	16	361
20	872¾	17	439
		18	514
	Height 19 feet.		
12	188½	9	56¾
13	242¾	10	81
14	304¼	11	111¼
15	374	12	147¼
16	452	13	189¾
17	537¾	14	237¾
18	631½	15	292¼
19	733	16	353
20	842	17	420
		18	493¾

STRENGTH OF STRUTS.

Oak.		Fir.	
Diameter in inches.	Breaking Weight in tons.	Diameter in inches.	Breaking Weight in tons.
Height 20 feet.			
12	177¼	10	75½
13	228¾	11	104
14	297½	12	138¾
15	347	13	178¾
16	431	14	225¼
17	514½	15	277½
18	606	16	336¾
19	705	17	402
20	812	18	473¼

CHAPTER X.

THEORY OF SHORING.

An experienced London architect, who is also a district surveyor, once expressed to the writer the opinion that shoring would probably never be reduced to a science. His meaning apparently was that common sense and practical experience are quite equal to the tasks imposed upon them in the ordinary operations of shoring, whereas, if scientific principles were called into play, much time would be occupied with complex calculations. It is certain that most men employ calculations only when they can save time by so doing, and that, in regard to temporary works, like shoring, it is scarcely worth while to economise closely in the strength of timber which can be used again and again. Still in shoring operations on a large scale, there are cases where a few rough calculations will conduce to the economy of labour, while theoretical principles may prove useful to students who have not the advantage of great experience.

The chief difficulty in calculations relative to shoring is to ascertain the amount of the thrusts to be resisted or provided against. There is no difficulty, however, in arriving at them approximately. The method of calculating the stability of walls and piers has been previously explained, and we have now to deal only with an extension of the same principle. If it requires a certain force, applied at a certain height, to overthrow a wall, an equal resistance applied in the opposite

direction will prevent it from falling. In ordinary cases, as the height at which a strut is placed against a wall is to the half-thickness of the wall, so is the weight of the wall to the required resistance of the strut. Thus, if we have to deal with a strip of 9 inch wall, 10 ft. long and 15 ft. high, which is in danger of falling over, it may, provided the joints are sound, be shored by means of a strut placed against it at a height of 10 ft. The weight of the wall, taken at 1 cwt. per cubic foot, will be $112\frac{1}{2}$ cwt., so that we have $\frac{112\frac{1}{2} \times 4\frac{1}{2}}{120} = 4\frac{1}{4}$ cwt., nearly, for the required resistance of the strut. Unless some extraneous pressure, such as wind force, be brought to bear upon the opposite side of the wall, the strain upon the strut cannot exceed the amount given; but, of course, an allowance should be made for the force of the wind, if there is any chance of its taking effect. An allowance of 40 lb. per foot for wind force would give about $53\frac{1}{2}$ cwts. upon the entire face of the wall. This may be considered to take effect at the centre of gravity of the wall-face, viz. at 7 feet 6 inches above the base of the wall. The effect of this, referred to a height of 10 feet, will be $\frac{53\frac{1}{2} \times 7\frac{1}{2}}{10} = 40$ cwt., nearly. The resistance of the strut should therefore be $44\frac{1}{4}$ cwt., and the strain upon it will depend upon its position, whether raking or perpendicular to the face of the wall. A raking shore generally rests upon the ground, flying shores being rarely placed in a raking position. The flying shore is generally to be preferred, because it is not dependent upon foundations for its security; but it should not be employed when there is any danger of its injuring the opposite building. When two walls or buildings incline together with equal force, flying shores

may advantageously be introduced between them; and the shores need be no stronger than if one of the buildings had been stationary. It is just as well to emphasise this simple truth, because it has been erroneously imagined that the shores must, in such a case, be of increased strength. Even so well-accepted an authority as C. Haden Stock, in his work on "Shoring and Underpinning," * favours this misapprehension in the following passage (page 14): "Of the two houses . . . the one on the left-hand is secure . . . . consequently the flying shore has to resist the thrust of the opposite house only. But when both are party walls it will be best to allow sufficient strength in the shore to resist the thrust of both the houses together," &c.

When a wall is to be sustained by means of a flying shore, it is obvious that the smallest amount of pressure requisite could be applied at the top. If placed too near the top, however, the resistance of the shore may have the effect of displacing the masonry at that point, while the portion below continues to move forward. There must therefore be a sufficient load of masonry above to resist lateral displacement. With a wall of uniform thickness, and without reckoning the resistance arising from the adhesive power of the mortar, calculations show that a flying shore might be placed very near indeed to the top of a wall—much nearer than it is ever placed in practice. This has reference to walls alone in which the thrust is proportioned to the weight. Considered as part of a building, however, a wall may be acted upon by thrusts which far exceed what would arise from its own weight, and which would make it dangerous to place a shore near the top.

* London: B. T. Batsford, 1882.

Before inquiring into the extents of thrusts that may arise from an entire building leaning over from the perpendicular, it may be stated that when a wall which is not bulged shows a disposition to turn over, hinging upon an open joint near the base, the best point for applying resistance is at the centre of oscillation. Hence it is advantageous to place a flying shore at about two-thirds of the height above the fulcrum, the chances of a bulge occurring above or below this point being reduced to a minimum.

To ascertain the thrust that may arise from a building which is leaning over, we must approximately estimate the weight of the building. A few main measurements will enable this to be done with sufficient accuracy for the purpose. The weight of the walls may be taken at 1 cwt. per cubic foot; the floors with their loads may be taken at $1\frac{1}{4}$ cwt. per square foot; and the surface of the roofing may be taken at 40 lb. per square foot, which will include the weight of all timbers with an allowance for wind force. The centre of gravity of the building being assumed to be upon its vertical centre line, the force necessary to overturn or sustain it may be calculated upon the same principle as when dealing with a wall or pier.

For example, if a tower 40 feet high and 10 feet in diameter weighs 100 tons, its moment of stability will be $100 \times 5 = 500$ foot-tons; and at a height of 30 feet it will require a force of $\frac{500}{30} = 16\frac{2}{3}$ tons to push it over or to prevent it from falling, assuming that the force is applied horizontally.

When several horizontal struts, or flying shores, are placed against a wall at different levels to prevent it from moving over, those which are placed lower should be stronger in the proportion that the thrust increases at

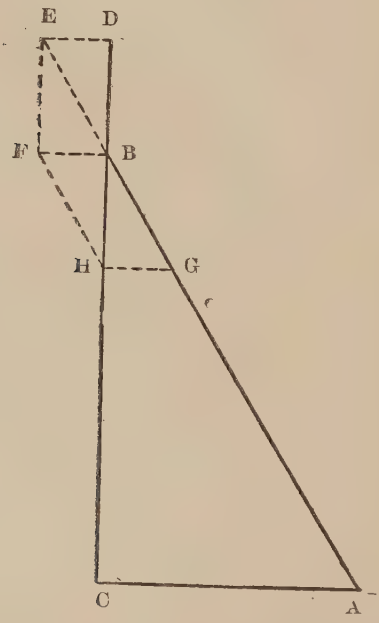
lower levels; and when there is a tendency to bulging in the wall, it is prudent to make each strut sufficiently strong to withstand the entire thrust independently. When the joints are sound, however, and there is no tendency to bulging, we may economise by providing struts of sufficient strength only to resist that share of the thrust that will fall to each. Thus in the case of the tower assumed above, strutting might be placed at heights of 10, 20, and 30 feet from the ground respectively. The entire thrust at each of those points, taken separately, would be 50 tons at 10 feet, 25 tons at 20 feet, and $16\frac{2}{3}$ tons at 30 feet. The strutting may then only be required to take one-third of the thrust at each level, namely, $16\frac{2}{3}$ tons at 10 feet, $8\frac{1}{3}$ tons at 20 feet, and $5\frac{5}{9}$ tons at 30 feet high.

We will now examine the conditions affecting raking shores as applied to resisting horizontal thrusts.

When a wall, which is moving over has to be shored from the ground, the shore should be so placed as to resist the horizontal thrust without heaving up the masonry above. In the accompanying diagram, let D C represent the face of a wall, which is about to be pushed over with a thrust F B, so that it will turn upon the point C. Let A B represent a raking shore, put up for the purpose of sustaining the wall. If the point B be fixed, and the wall move forward, the point A will tend to slide outwards in the direction C A, with a thrust equal to F B. Draw F H, parallel to B A, cutting B C in H, and draw H G, parallel to C A, cutting B A in G. Then H G is equal to the thrust F B, and B G is proportioned to the compressive strain upon the shore B A. If the point A be fixed, and the point B free, then, when the wall moves forward, B will tend to slide up the wall with a thrust proportioned to H B. When the head of

the shore B is fixed by means of a needle or stud, it is necessary that the weight D B shall be at least equal to the upward thrust H B. Let D B represent the weight of this upper portion of the wall. Draw E D parallel and equal to F B, and E F parallel and equal to D B, and join E B. Then E B represents the resultant

Fig. 16.



of these two forces, D B the weight, and F B the thrust. Now since it is necessary that H B should not exceed D B, it follows that the slope of the shore must not be steeper than E B, otherwise it will heave up the upper portion of the wall.

If we cannot conveniently get sufficient weight above B, and cannot place the foot of the shore further out from the wall, we must continue the wall-piece some distance below B, and secure it there with an additional needle or stud.

The weight of the shore takes effect in the following manner. One-half the weight acts vertically at A, and exerts a horizontal thrust in the direction C A, which thrust bears the same proportion to the half weight as C A does to B C. The other half of the weight acts vertically down B C, and exerts a proportionate thrust in the direction B F. The effect is to neutralise a portion of the thrust F B, and consequently to reduce the weight D B, which is required to produce equilibrium. The ratio of F B to D B therefore remains unaltered.

It is possible that cases might occur where it would be convenient to increase the weight of shoring in order to counteract opposing thrusts. It is evident that a weight suspended from B would produce a thrust in the direction B F, which would bear the same proportion to the weight as B C bears to C A.

The condition of a house front will often necessitate placing the head of a shore near the top of the wall, while the narrowness of the street footway will prevent the foot of the shore from being placed as far out as is desirable. Under such circumstances, many shores are, in practice, so placed that the top raker could not, as a single strut, exert its full power in sustaining the wall without forcing up the brickwork above, and thereby defeating the object of shoring. The necessity for more than one raker in a single system or battery of shores is then apparent, quite apart from any tendency to bulging that may exist at any point in the height of the wall. The best points at which to fix the heads of the rakers

will differ in different cases. The heads of the rakers should be ordinarily at the floor levels, where they will prevent the joists from pushing out the wall, and the joists in their turn will prevent the shores from pushing the wall in. When the joists run crosswise, however, the circumstances differ, and more dependence is placed upon the rigidity of the wall-piece, and its effect in distributing the resistance of the shore, and neutralising its tendency to push in the wall. When the latter is at all likely to occur, the need for internal strutting from wall to wall becomes apparent. The head of a top-raker should be placed no higher than will permit a line in continuation of its upper surface to fall within the section of the roof.

We will now examine the mechanical conditions which indicate the most advantageous positions for the heads of the separate rakers in a system of shores, apart from the question of floor-thrusts or the resistance afforded by party walls.

Let us assume that a system of raking shores has to be applied to a portion of a structure 30 feet high above the street, tending to lean forward with a thrust equivalent to one ton at the top. We will assume that the curb of the footway is only 5 feet away from the wall, that the shoring must not encroach upon the roadway, and that the head of the top raker must be placed no lower than 5 feet from the wall, i. e. 25 feet above the street level. The moment of the thrust at the top of the wall is 30 foot-tons, and its effect at 25 feet high is therefore $\frac{30}{25} = 1\frac{1}{5}$ ton = 24 cwt. We want to know whether there will be a sufficient load of masonry above the shore to resist its vertical thrust. This will depend upon the condition of the wall, and whether or not a narrow strip of masonry is liable to be pushed upwards, breaking its

connection with the masonry on each side of it—a highly dangerous contingency. We will assume that the vertical resistance of the wall is equal to 5 cwt. for every vertical foot of height, bearing in mind that it would not be safe to allow so much as this in respect of a parapet above the roof. We have now a horizontal thrust of 24 cwt. and a vertical pressure of 25 cwt. impinging against the head of our shore. It is evident that the resultant of these two forces inclines at an angle very little steeper than 45°, and as the shore is very much steeper than this, it cannot resist the horizontal thrust without overcoming the vertical pressure, and so heaving up the masonry above. This shore must only be called upon to resist a portion of the thrust whose resultant coincides with the slope of the shore; the remainder of the thrust must be resisted by another shore placed lower down.

The shore, the footway, and the wall together form a right-angled triangle, of which the base is to the perpendicular as 1 to 5. The forces resisted by the shore must be in the same proportion, if their resultant is to coincide with its slope; that is to say, the horizontal thrust must be to the vertical pressure as 1 to 5, and since the vertical pressure is 25 cwt., we have 5 cwt. for the thrust resisted by the shore, leaving another 19 cwt. unresisted. The moment of this surplus thrust is $19 \times 25 = 475$ foot-cwts.

We have now to find the height at which the second shore may most economically be placed. We will assume that the two shores are to have their feet bound together, as is ordinarily the case. The foot of the second shore will, therefore, be 5 feet from the base of the wall, and its head will be at a height not yet determined, which we will call x, assuming it to be

expressed in feet. It will have to resist a horizontal thrust which we will call t, and will be kept down by a vertical pressure p, due to the weight of the masonry below the upper shore. The values of t and p are to be expressed in cwts. In order, then, that the slope of this lower shore may coincide with the resultant of t and p, the following proportion must be maintained :—

As $5 : x :: t : p$, and $\frac{x\,t}{5} = p$. But $x\,t$ is the moment of t, which is of course equal to 475, so that $p = \frac{475}{5} = 95$ cwt. ; and as the vertical weight is assumed at 5 cwt. per foot, we have $\frac{9.5}{5} = 19$ feet for the distance of the head of the lower shore from that of the upper one, the height x being = 6 feet. Subject to the above conditions, this is the most economical position for the lower shore.

It remains, in concluding these fragmentary notes upon a subject which would admit of more exhaustive treatment, to remind the reader of the very simple means by which the required scantlings of the shores can be ascertained, when the extent of the thrusts acting upon them is known. The resultant of t and p, for instance, in the case assumed above is $= \sqrt{(t^2 + p^2)}$; and this represents the stress upon the shore. The required scantling may be ascertained by means of the formula previously given for struts, shores being usually of fir. As regards the factor of safety, $\frac{1}{10}$ is usual for permanent struts of wood, but for temporary work like shoring $\frac{1}{6}$ is sufficient, unless the work is to remain for a long time exposed to wet weather, in which case $\frac{1}{8}$ is to be recommended. A good margin of strength is obtained when

the shores are stiffened by means of cross-strutting, and when this is thoroughly done, the unsupported lengths of timber being reduced to about 6 feet, with scantlings not less than 6 inches by 6 inches, the safe stress upon the timber may be taken at between 5 cwt. and 6 cwt. per square inch of sectional area.

THE END.

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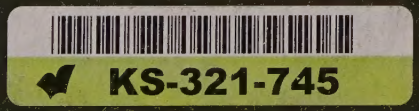
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